Backwards Problem in Geotechnical Earthquake Engineering

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ABSTRACT: The authors initial thought on the backwards problem in geotechnical earthquake engineering is presented through an example of damage to caisson quaywall during earthquakes. Both a simplified and detailed dynamic analyses are presented. It is essential to confirm, at the outset, that the backwards problem.is well defined. There should be sufficient geotechnical data and earthquake data to match the analysis procedure used for solving the backwards problem. Ill-defined backwards problem, either due to lack of required geotechnical or earthquake motion data, should be corrected before the trial solution of the backwards problem.

1 INTRODUCTON

Backwards problem in geotechnical earthquake engineering is to identify the process to the residual states (often in terms of excessive deformation rather than complete collapse) of geotechnical structures as the most likely scenario that is explained through geotechnical and earthquake data. Backwards problem usually consists of three steps. The first step is to define the problem in terms of failure mode and extent of deformation. If the problem involves the function of the geotechnical structure, then the degree of damage to the function (indirect damage) should also be defined in terms of performance objectives. The second step is to assume possible processes of scenarios from the initial state (before the earthquake) to the residual state (after the earthquake). The final step is to select the most likely scenario(s) based available data from geotechnical investigations and earthquake motion recording.

This paper presents the author's initial thought on the backwards problem in geotechnical earthquake engineering using the example of a caisson type quaywall, a relatively simple soil-structure system.

2 SIMPLIFIED DYNAMICS ANALYSIS

A gravity quaywall is made of a caisson or other rigid wall put on a seabed, and maintains its stability by a friction at the bottom of the wall. Typical failure modes during earthquakes involve a seaward displacement, settlement and a tilt. For a quaywall constructed on a firm foundation, an increase in the earth pressure from the backfill plus the effect of an inertia force on the body of the wall result in the seaward movement of the wall as shown in Fig. 1(a). If a width to height ratio of the wall is small, a tilt may also be involved. The past case histories of gravity quaywalls often belong to this category (e.g. Iai, 1998). When the subsoil below the gravity wall is loose and excess pore water pressure is increased in the subsoil, however, the movement of the wall is presumed to be associated with a significant deformation in the foundation soil, resulting in a large seaward movement involving a tilt and settlement as shown in Fig. 1(b).

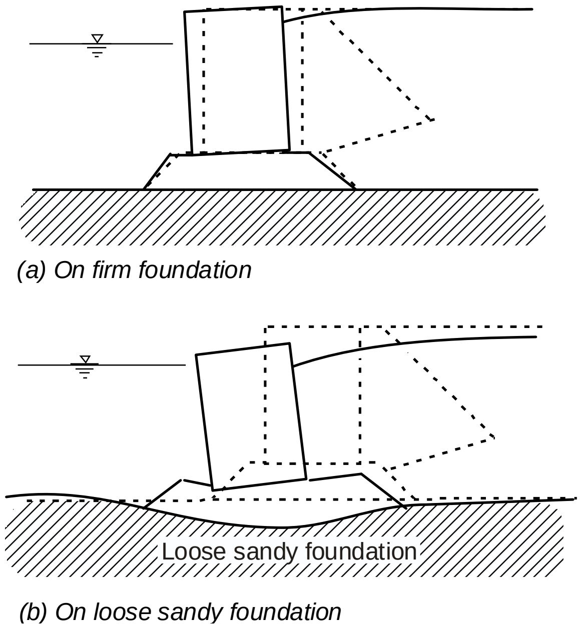

Fig. 1 Deformation/failure modes of gravity quaywall

The former mode of failure, shown in Fig. 1(a), has been conventionally analyzed by evaluating the seismic active earth pressures using Mononobe-Okabe equation (Mononobe, 1924; Okabe, 1924). This equation is derived by modifying Coulomb's classical earth pressure theory to account for inertia forces. In the uniform field of horizontal and (downward) vertical accelerations, $k_h g$ and $k_v g$, the body force vector, originally pointing downward due to gravity, is rotated by the seismic inertia angle, ψ, defined by (see Fig.2)

$$\psi = \tan^{-1}\left[\frac{k_h}{1-k_v}\right] \tag{1}$$

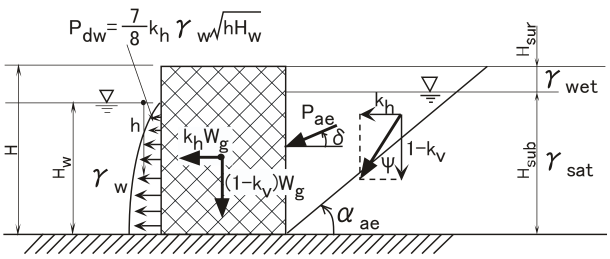

Fig. 2 Active earth pressure and other actions on a gravity caisson during earthquakes

The Mononobe-Okabe equation is, thus, obtained by rotating the geometry of Coulomb's classical solution through the seismic inertia angle, ψ, and scaling the magnitude of the body force to fit the resultant of the gravity and the inertia forces.

Simple, straightforward methods have been developed for evaluating the permanent displacements of a sliding block (Newmark, 1965). In this analysis, first, the stability of the wall and the backfill are evaluated using the lateral earth pressure theory based on Mononobe-Okabe's equations. The threshold acceleration is determined by the value resulting in a factor of safety of unity for sliding of the wall-backfill system. For example, the threshold acceleration, at, for a vertical retaining wall is given by the expression (Richards and Elms, 1979)

$$a_t = \left(\mu_b - \frac{P_{ae}\cos\delta - \mu_b \sin\delta}{W_g}\right)g \tag{2}$$

where μ_b is the coefficient of interface friction between the wall and the foundation rubble or soil, P_{ae} is the active earth thrust computed using the Mononobe-Okabe method, δ is the wall-backfill interface friction angle, W_g is the weight of the wall per unit width, and g is the acceleration of gravity. It should be noted that a_t must be known in order to calculate P_{ae}, therefore an iterative procedure is necessary.

Once the threshold acceleration has been determined, then a set of acceleration time histories are selected for sliding block analysis. Displacements calculated through sliding block analysis are sensitive to the characteristics of acceleration tie history used in the analysis. The acceleration time histories should be representative of the seismic condition in both duration and frequency content. When the ground motion acceleration exceeds the threshold acceleration, a_t, the wall-backfill system begins to move by translation along the base of the wall and the failure plane through the backfill. By double integrating the area of the acceleration time history that exceeds a_t, and continuing the time integration until the sliding stops, the displacement of the wall relative to the firm base below the failure plane can be determined as shown in Fig. 3. This computation can be easily performed using common spreadsheet routines or a simple computer code.

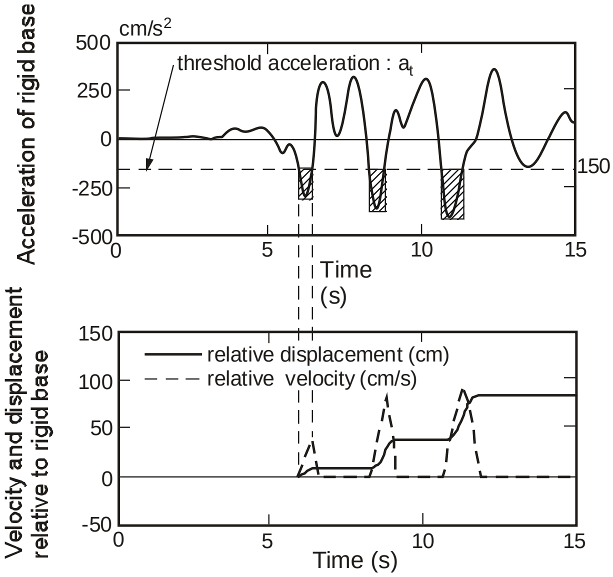

Fig. 3 Example of computing displacement in the sliding block analysis

3 DETAILED DYNAMIC ANALYSIS

The latter mode of failure, shown in Fig. 1(b), had not received wide attention until the Great Hanshin earthquake of 1995. Many of the caisson walls in Kobe port were constructed on a loose saturated backfill foundation of decomposed granite, which was used for replacing the soft clayey deposit in Kobe Port to attain the required bearing capacity of foundation. Shaken with a strong earthquake motion having the peak accelerations of 0.54g and

0.45g in the horizontal and vertical directions, these caisson walls displaced toward the sea about 5 m maximum, 3m on average, settled 1 to 2 m and tilted about 4 degrees toward the sea. Figure 4 shows a typical example of the cross section and the deformation after the earthquake (Inagaki et al., 1996). Although the sliding mechanism could explain the large horizontal displacement of the caisson walls, this mechanism did not explain the large settlement and tilt of the caissons. Reduction in the stiffness of foundation soils due to an excess pore water pressure increase, then, was speculated as a main cause of the damage to the caisson walls at Kobe Port.

This speculation was confirmed by a series of effective stress analyses based on the strain space multiple mechanism model (Iai et al., 1998). The model parameters were evaluated based on the in-situ velocity logging, the blow counts of Standard Penetration Tests (SPT N-values) and the results of the cyclic triaxial tests. The specimens used for the cyclic triaxial tests were undisturbed samples 60 cm long with a diameter of 30 cm obtained by in-situ freezing technique. The input earthquake motions were those at the Port Island site successfully recorded by the Kobe City Government. The analysis domain used for the finite element analysis covered a cross sectional area of about 220 m by 40 m in the horizontal and vertical directions.

The effective stress analysis resulted in the residual deformation shown in Fig. 5. As shown in this figure, the mode of deformation of the caisson wall was to tilt into and push out the foundation soil beneath the caisson. This was consistent with the observed deformation mode of the rubble foundation shown in Fig. 6, which was investigated by divers. Note that a significant deformation was induced in the foundation soil beneath the caisson wall. The order of displacements of the wall was also comparable to that observed and shown in Fig. 4.

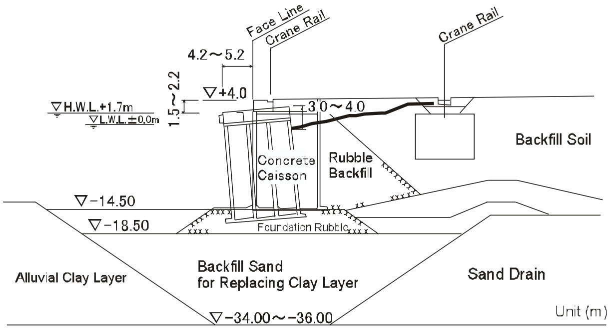

Fig. 4 Cross section of gravity quay wall at Kobe port and deformation/failure during 1995 Great Hanshin earthquake, Japan

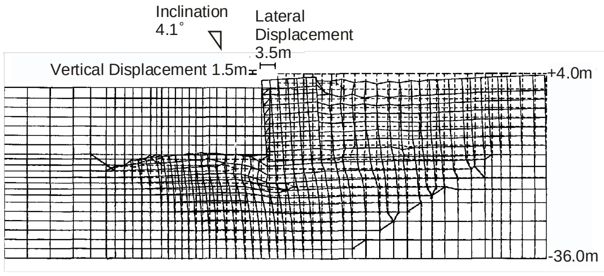

Fig. 5 Computed deformation of a gravity quaywall

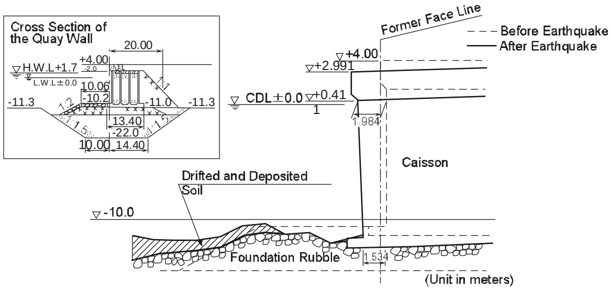

Fig. 6 Deformation of rubble foundation of a quay wall investigated by divers

In order to evaluate the overall effect of geotechnical conditions on the displacements of a gravity wall, the following three analyses were performed as a parameter study. The parameter study included a virtual soil model, to be called non-liquefiable soil, which has the same properties as those used in the aforementioned analysis but without the effect of dilatancy. To distinguish the cases in the parameter study, the case which dealt with the actual quaywall during the earthquake described earlier is designated as Case-1.

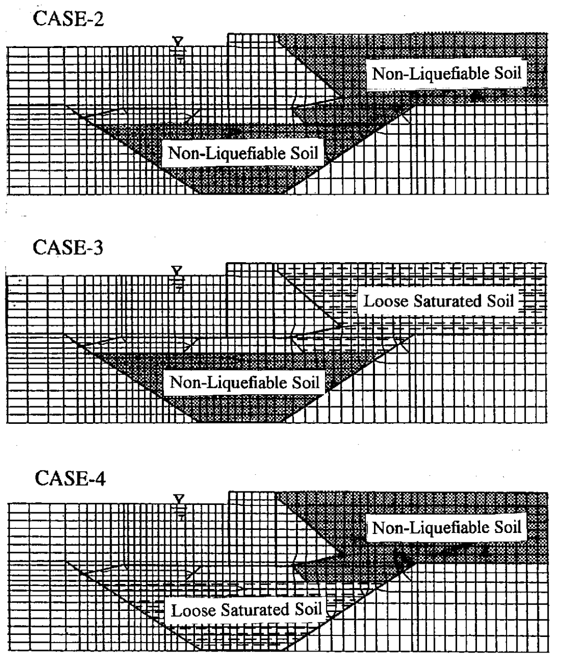

Fig. 7 Conditions assumed for parametric study, Cases 2 through 4

Cases-2 through 4 are defined depending on the extent of the non-liquefiable soil relative to the caisson wall as shown in Fig. 7.

The major results of the analysis are summarized in Table 1 including those of Case-1. These results indicate that the deformation of the gravity wall may be reduced up to about one half of that at the earthquake if the excess pore water pressure increase were prevented in the subsoil as Case-2. In particular, the effect of the pore water pressure increase in the foundation soil beneath the caisson wall is about twice as that of the backfill as understood from the comparison of Cases-3 and 4. These results were partly confirmed by the seismic performance of the quay walls at Port Island (phase II), where a caisson wall had been put on the foundation improved with a sand compaction pile technique at the time of the earthquake (Iai et al., 1998).

For variations in the peak acceleration of the earthquake motion input at the base, the horizontal residual displacement at the top of the caisson wall was computed as shown in Fig. 8. These response curves constitute the basis for performance based design. For example, let us suppose a caisson quay wall is constructed at a site where seismic hazard analysis resulted in 0.2g and 0.3g for Levels 1 and 2 earthquakes, respectively.

Table 1 Major results of parametric study for gravity quay wall

Case	Residual Displacements of Caisson Horizontal (m)	Vertical (m)	Tilt (degrees)
Case 1	3.5	1.5	4.1
Case 2	1.6	0.6	2.4
Case 3	2.1	0.7	3.1
Case 4	2.5	1.1	2.2

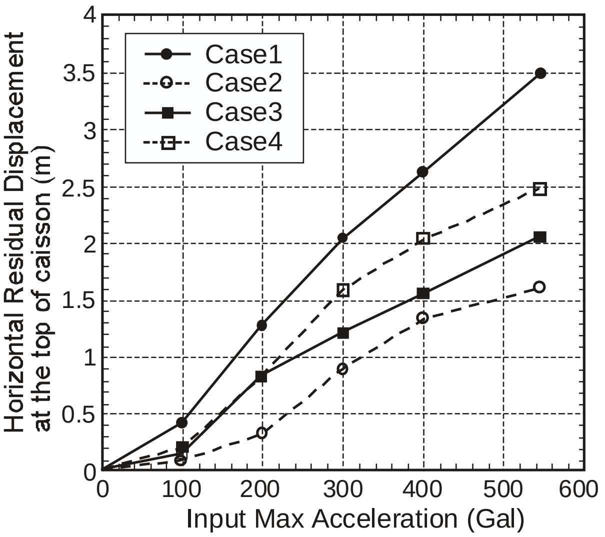

Fig. 8 Effects of input acceleration levels on horizontal residual displacement

For a Case 1 quay wall, horizontal displacements at the top of the caisson will be 1.3 and 2.1 m for Levels 1 and 2 earthquakes.

For a Case 2 quay wall with full liquefaction remediation measures, displacements will be reduced to 0.3 and 0.9 m for Levels 1 and 2 earthquakes, which may satisfy the performance criteria with respect to allowable displacement. The effect of the vertical component of input acceleration time histories was studied by varying the peak acceleration of the vertical component of acceleration ranging from zero to 0.40g (Ichii et al., 1997). The peak acceleration of the horizontal component was unchanged from the original value of 0.54g. The phase difference between the vertical and the horizontal components were also varied. This parametric study suggested that the effect of the vertical component is to increase the residual displacement less than about 10 percent.

As discussed in this section, geotechnical conditions significantly affect the performance of a gravity quaywall. They govern the mode and extent of the deformation/failure. In particular, the failure mode associated with a loose foundation soil should be more thoroughly evaluated in the design practice. Appropriate characterization of these conditions and a suitable seismic analysis will lead to a reasonable seismic design of a gravity quaywall.

4 BACKWARDS PROBLEM THROUGH SIMPLIFIED DYNAMIC ANALYSIS

If a caisson quaywall is constructed on firm foundation and if the failure mode of the caisson quaywall is confirmed in translational sliding mode. then the backwards problem becomes relatively simple. The crucial steps in this simple backwards problem can be reduced to evaluate the threshold acceleration for sliding defined by Eq. (2) and to evaluate the acceleration time history during the earthquake. As shown in Eqs. (1) and (2), geotechnical parameters that should be evaluated are internal friction angle of backfill, the coefficient of interface friction between the wall and the foundation rubble or soil, and the wall-backfill interface friction angle.

In practice, the acceleration time history during the earthquake is often missing. In this unfortunate case, a large uncertainty will be left in the backwards problem even in this simplest backwards problem.

5 BACKWARDS PROBLEM THROUGH DETAILED DYNAMIC ANALYSIS

If a caisson quaywall is constructed on loose sandy foundation, then the backwards problem needs detailed dynamic analysis by taking account geotechnical conditions of backfill and foundation soil. The crucial steps in this backwards problem follows the procedure in solving the typical boundary value problem: (1) initial condition, (2) boundary condition, (3) input earthquake motions, (4) appropriate constitutive modeling of soil, structure, and interface element, (5) appropriate numerical solution procedure through the use of finite element technique.

Initial condition for a soil-structure system in geotechnical earthquake engineering is often computed through a static analysis using the same mesh and constitutive equation for dynamic analysis but by applying gravity acceleration. Boundary conditions should take into account the transmission of the seismic wave in and out from the analysis domain for finite element analysis. The constitutive equation is utmost importance and the applicability of this equation for various loading conditions should be pre-confirmed.

If all the above procedures are satisfied, then the essential problem in backwards problem becomes the procedure to evaluate the geotechnical conditions. In the example presented earlier for the caisson wall in Kobe Port, the model parameters were evaluated based on the in-situ velocity logging, the blow counts of Standard Penetration Tests (SPT N-values) and the results of the cyclic triaxial tests. The specimens used for the cyclic triaxial tests were undisturbed samples 60 cm long with a diameter of 30 cm obtained by in-situ freezing technique. These sets of procedures actually performed are considered the most ideal case. As a result, the primary cause of the damage to the caisson quaywall was identified as shown earlier. In particular, the summary of the parameter study shown in Table 1 indicate that inertia effect is about 50% (=1.6m/3.5m) (Case-2), and the effect of the liquefaction in foundation is about 30%(=(2.5m-1.6m)/3.5m) (Case-4) and the rest is due to the effect of the liquefaction at backfill (Case-3). The liquefaction of backfill and foundation soil (Case-1 and 3) also takes account of the primary cause of tilting.

In practice, only crude geotechnical investigation may be possible. In this practical case, there will be large uncertainty in geotechnical conditions. Availability of the earthquake ground motion is another matter of question in practice as discussed earlier. It might be possible to arrive at the correct answer by putting the two wrong answers together, one with wrong geotechnical condition, the other with wrong earthquake motion data. In this case, the backwards problem is ill-defined. The backwards problem has to be correctly defined at first, then the efforts to solve this problem will have a chance to give us a correct answer and contribute to enhancing our engineering knowledge in geotechnical earthquake engineering.

6 CONCLUSIONS

The authors initial thought on the backwards problem in geotechnical earthquake engineering is presented through an example of caisson type quaywalls during earthquakes. Both the simplified and detailed dynamic analyses are presented.

Backwards problem in geotechnical earthquake engineering is to identify the process to the residual states (often in terms of excessive deformation rather than complete collapse) of geotechnical structures as the most likely scenario that is explained through geotechnical and earthquake data. Backwards problem usually consists of three steps. The first step is to define the problem in terms of failure mode and extent of deformation. If the problem involves the function of the geotechnical structure, then the degree of damage to the function (indirect damage) should also be defined in terms of performance objectives. The second step is to assume possible processes of scenarios from the initial state (before the earthquake) to the residual state (after the earthquake). The final step is to select the most likely scenario(s) based available data from geotechnical investigations and earthquake motion recording.

It is essential to confirm, at the outset, that the backwards problem.is well defined. Ill-defined backwards problem, either due to lack of required geotechnical or earthquake motion data, should be corrected before the efforts are put into the solution of the backwards problem.

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